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THE TAVETSCH INTERMEDIATE MASSIF NORTH: PROOF OF FEASIBILITY

Heinz Ehrbar

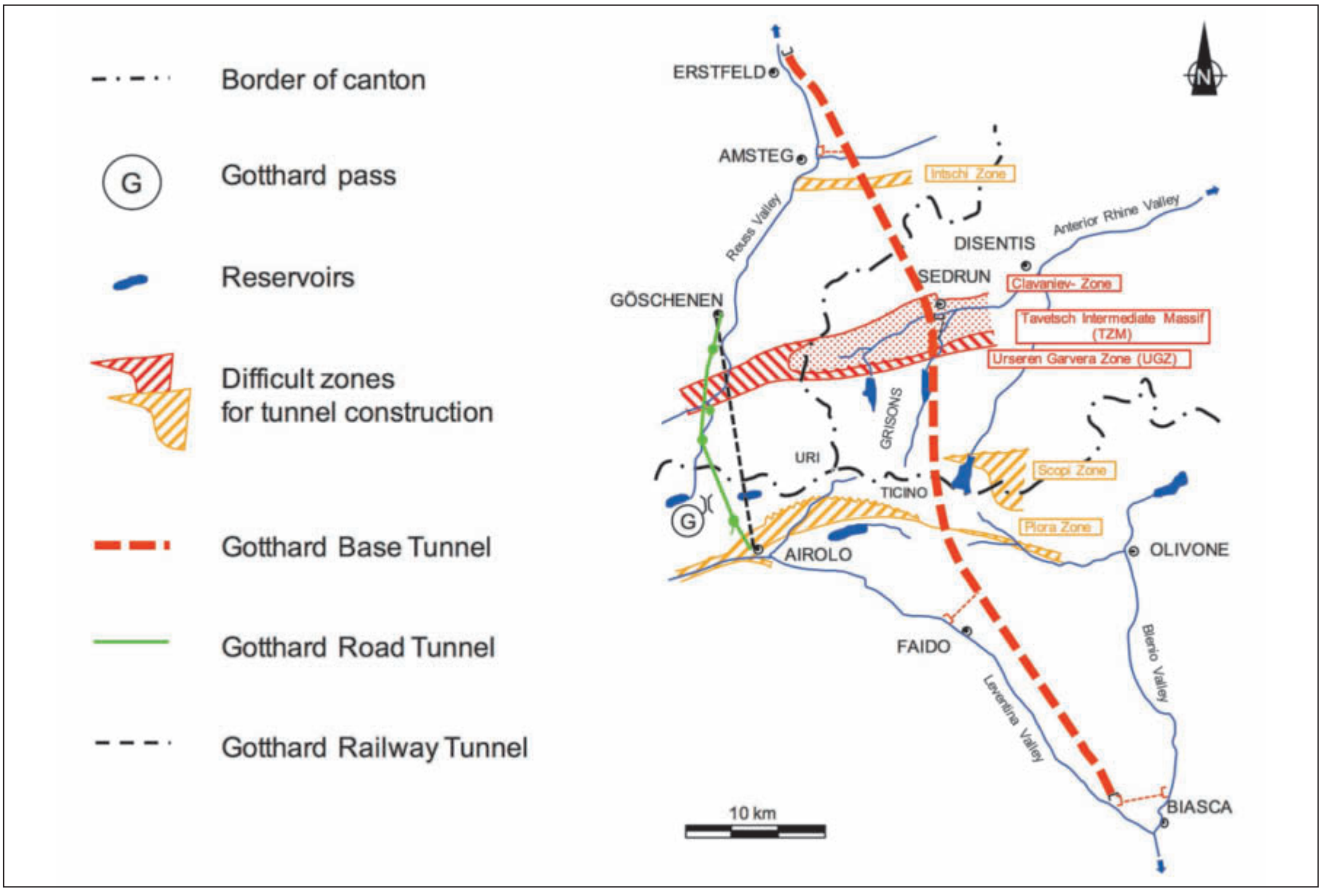
1 INITIAL SITUATION

The federal decree of October 1991 on the construction of the Swiss rail link through the Alps (the Alptransit decree) approved the construction of the Gotthard Base Tunnel (GBT) between Erstfeld and Bodio in principle. The many constraints resulted in only low degrees of freedom for the routing of the "corridor" selected for the GBT. An essential criterion for the alignment of the GBT was the avoidance of zones with difficulties for tunnel construction or the crossing of them only for the shortest-possible distance.

The Tavetsch intermediate massif and its northern zone in particular, the subsequent Clavaniev zone, and the Urseren-Garvera zone, known from construction of the Gotthard road tunnel, were identified as critical zones in the vicinity of Sedrun.

The Tavetsch intermediate massif north was severely tectonically stressed during the folding of the Alps, the originally compact rock strata being degraded to an almost loose-rock-type ground. Even in the earliest federal studies into the construction of a Gotthard base tunnel, the hazard of the occurrence of squeezing rock conditions in the Tavetsch intermediate massif, which would need to be tunnelled at a rate of advance of approximately 1.8 m/day, was described already in 1963 [1]. The SBB's 1975 project for the construction of a base tunnel then envisaged a double-track tunnel to be created at a daily advance rate of 3 m [2].

The "Yes" vote in the 1992 referendum on the Alptransit decree set the political path for implementation of the AlpTransit project firmly in favour of the Gotthard and Lötschberg routes. The drafting phase for the preliminary project then began. The feasibility of a section of tunnel in the Tavetsch intermediate massif was, during this phase, considered by many to be just as critical as tunnelling in the Piora syncline, due to the difficult squeezing rock phenomenon anticipated. All those responsible for the project were fully aware from the inception that, on the one hand, a double-track tunnel had to be excluded here and, on the other hand, that an intermediate access point would be



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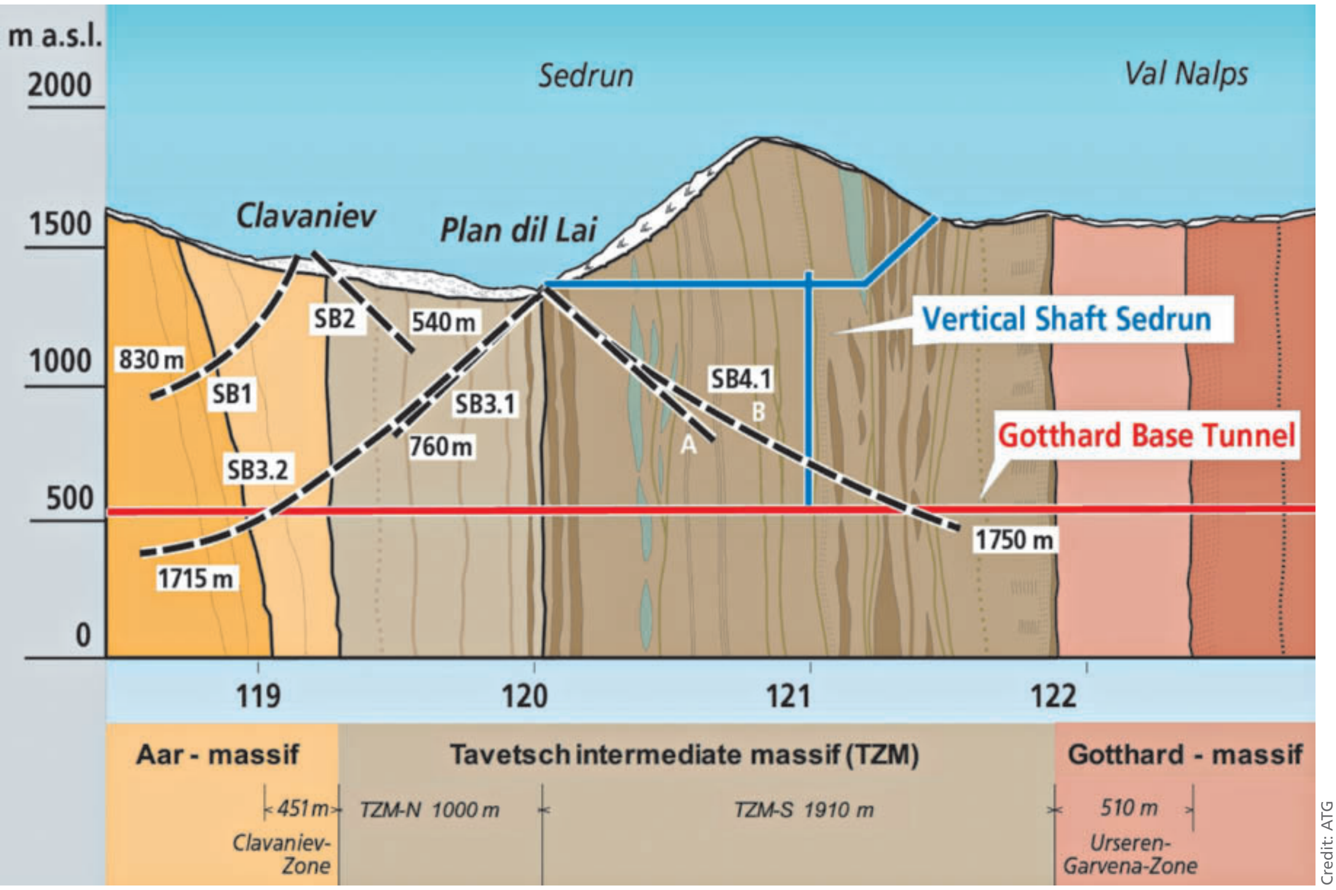
▸ **Fig. 1** *The GBT alignment and zones of structural difficulty*



▶ **Fig. 2** *Tectonic units in the vicinity of Sedrun, showing exploratory boreholes*

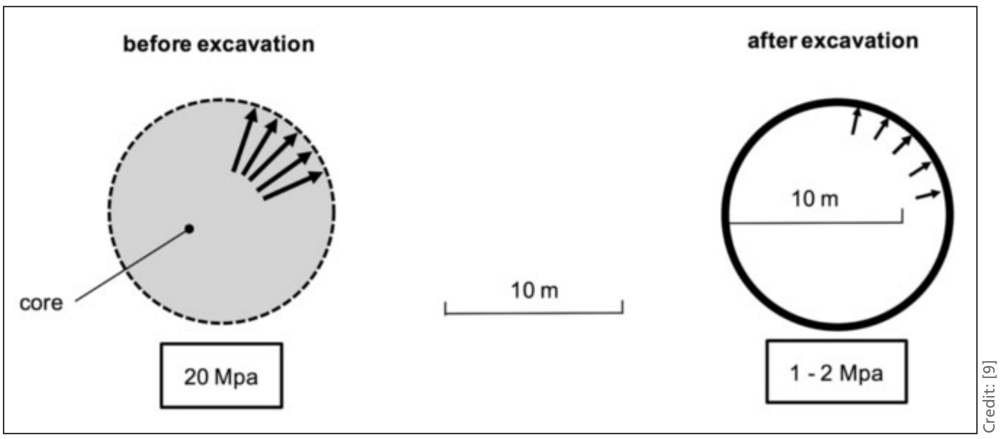
needed in the vicinity – in view of the anticipated low rates of advance – in order to keep overall construction time within the planned window.

The knowledge necessary for engineering assessment of the task in hand was obtained by means of three different exploratory borehole campaigns (1990, 1993 and 1997/98) [4]. The results of drilling and the in situ and the laboratory tests confirmed that it would be necessary to anticipate highly heterogeneous and exceptionally demanding tunnel-engineering conditions. The percentage of highly fault-gouged and softer rock was estimated at 71 % for the around 1,150 m long zone in the Tavetsch intermediate massif north, while the remaining 29 % would be hard gneiss formations as intermediate strata, with brittle fracture behaviour. The ground behaviour of this "sandwich structure" during construction could be forecast only with difficulty.

Account was taken of this fact in modelling of the underground conditions by defining zones of differing homogeneity, which were differentiated by differing intensity and combination of the anticipated hazard scenarios of "genuine rock pressure" and "face instability", and by the possible influence of pore-water pressures [7].

2 CLARIFICATION OF FEASIBILITY FOR A UNIQUE ASSIGNMENT

There were, on the basis of experience from other projects with comparable geotechnical conditions, reservations concerning the technical feasibility of tunnelling through the Tavetsch



▶ **Fig. 3** *The task of designing the excavation support system* intermediate massif north. Tunnelling in the Mesozoic of the Gotthard road tunnel, for which only a cover of 300 m had to be overcome, however, was examined for the comparative analyses, in particular; other projects in squeezing rock were also correspondingly analysed, however [2].

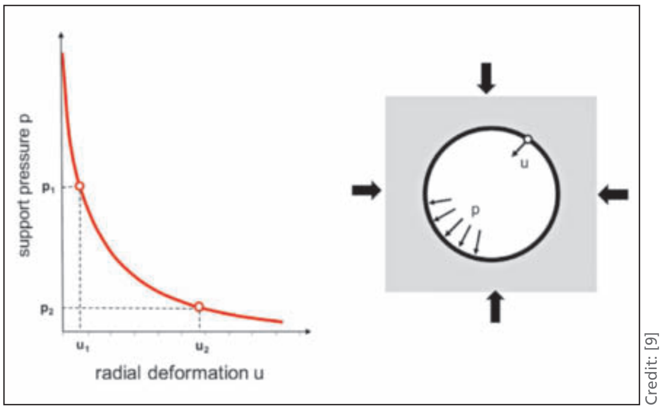
It was, in fact, not possible to base deliberations on any excavation support concept or on any construction and operation procedures under analogous boundary conditions encountered up to now. Concepts which were required to satisfy both the rock-structural and the operational construction requirements were developed on the basis of close coordination between the project engineer and the Construction Technology Working Group, a team of experts appointed by the client.

An answer was needed to the following question: is it possible to establish, and to maintain for an operational period of 100 years, a new equilibrium in a tunnel with an excavated diameter of up to 13 m by applying a support resistance of approximately maximum 2 MPa (the assumed technical and economic maximum for the given boundary conditions) if initial stresses of around 20 MPa predominate in the virgin rock structure (see ▶ **Fig. 3**)?

The characteristic-curve method served primarily as the basis for structural investigations of the tunnel during the draft phase. The ground characteristic curve represents, under a simplified assumption, the correlation between support resistance and the radial displacement at the excavation boundary. This correlation is non-linear in an elastic-plastic material. The most important statement on the characteristic curve, unequivocally confirmed by observations, is that the support resistance necessary for equilibrium decreases as displacement increases (compare the pairs u1/p1 and u2/p2 in ▶ **Fig. 4**).

This fundamental context makes it possible to formulate two boundary cases for the tunnel design in squeezing rock [5]:

1. The resistance principle
2. The yielding principle

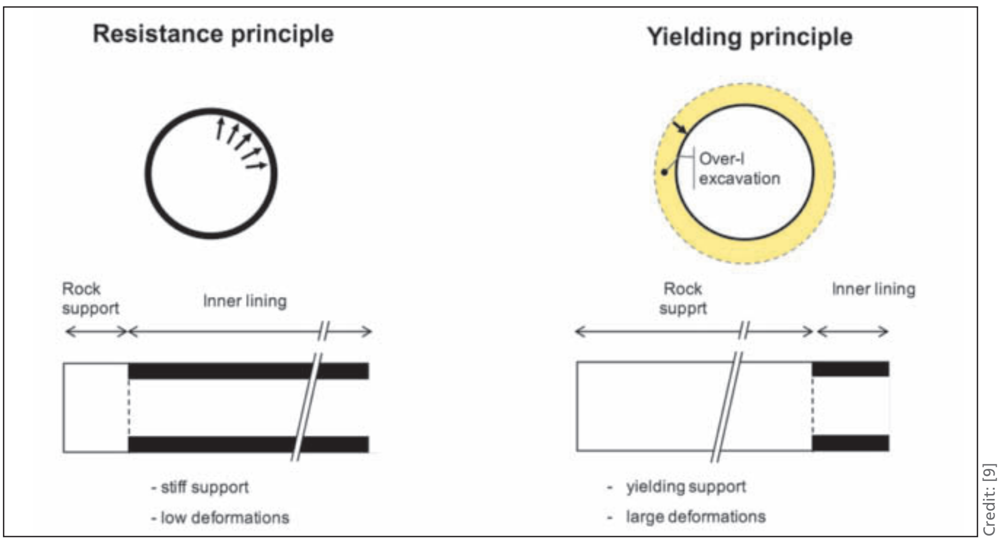


▶ **Fig. 4** *The characteristic curve*

In the former case, the emphasis is on adequate support of the cavity using rigid support with the lowest-possible deformations. A lower support resistance is required for the attainment of equilibrium in the latter case, since, due to the greater amount of excavation, deformations can be permitted, necessitating yielding excavation support.

The two principles also differ in terms of procedure in the longitudinal direction of the tunnel: in the case of the resistance principle using full-face excavation, excavation support is accomplished using heavy-duty steel sets. Rock deformations here remain relatively slight. The interior tunnel lining, with a higher load-bearing capability, is generally continued up-close to the face when the resistance principle is used.

In the yielding principle, a larger excavation profile creates space for the convergences anticipated. The excavation support must then be correspondingly yielding. Installation of the rigid interior tunnel lining can then be accomplished at great spatial and chronological intervals from the face, and even months or years afterward, when the deformations have subsided.



▶ **Fig. 5** *Fundamental principles of excavation support*

The resistance principle was used successfully in the construction of the tunnels for the Altà Velocità rail link between Bologna and Florence in the 1990s [6]. Full-face excavation was utilised with excavation areas of 100–120 m² and the typical face bolts. This tunnelling method was, at the time, a revolutionary innovation.

For the project engineer and for the Construction Technology Working Group intensively participating in the project, the question arose in

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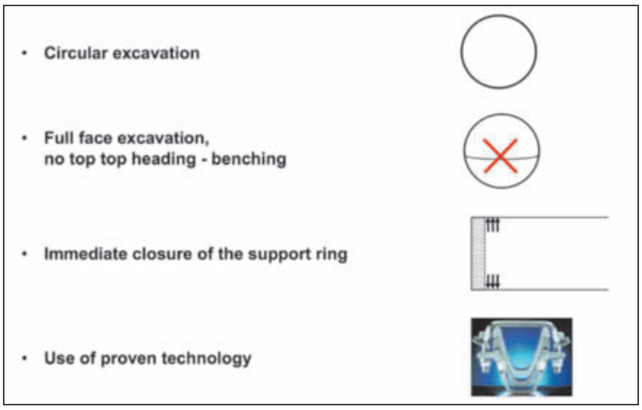
▶ **Fig. 6** *Example of tunnelling on the resistance principle*



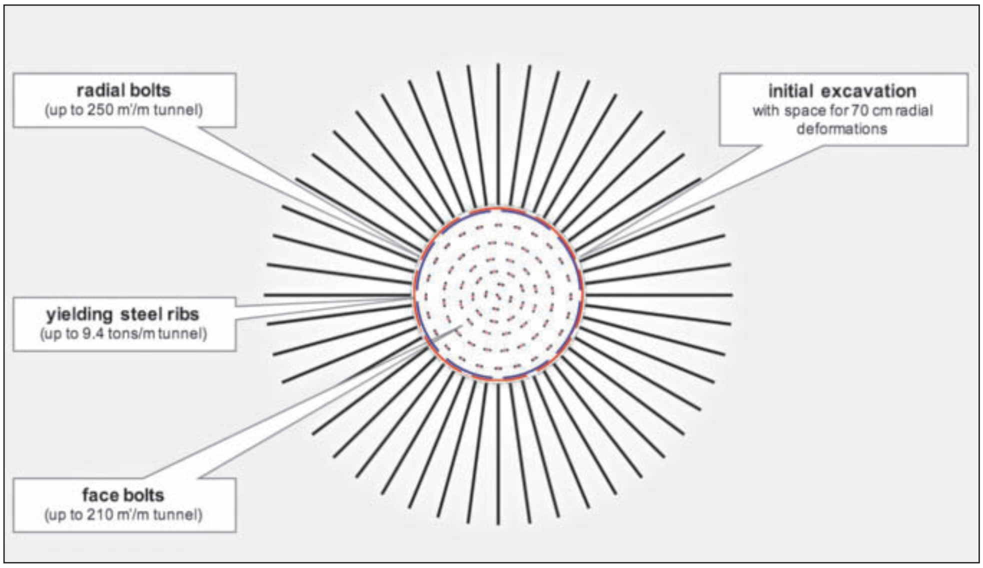
Credit: [9]

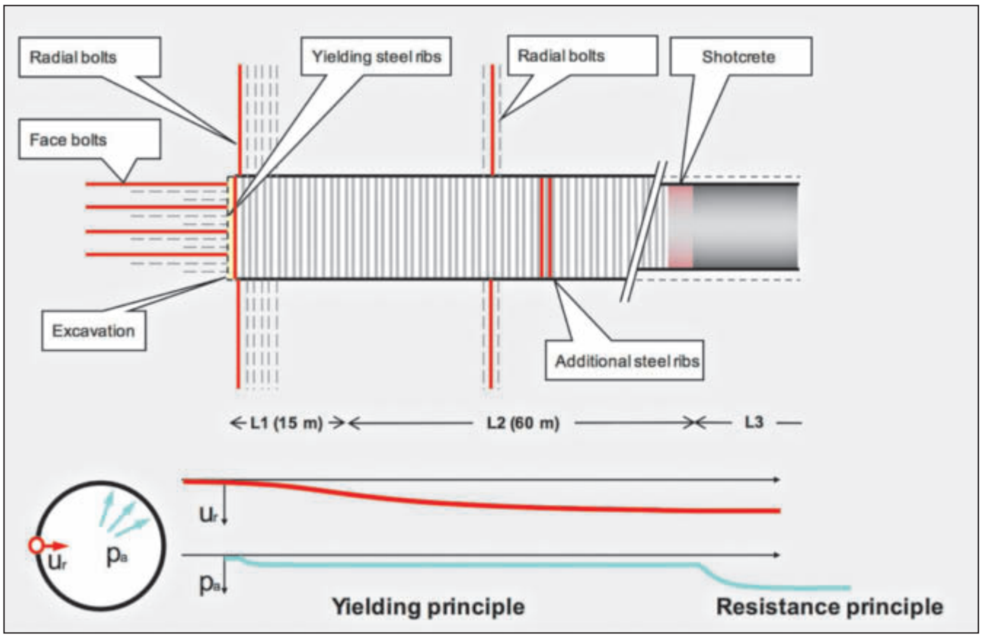
▶ **Fig. 7** *Boundary conditions for the design of the excavation support*

planning the Tavetsch intermediate massif north section of tunnel of whether the full-face excavation method using the resistance principle, tried and proven with great success in Italy, could be adopted here. It had to be borne in mind that there, under otherwise similar geotechnical conditions to those in the Tavetsch intermediate massif north, the rock overburden had reached only 200–300 m, and 500 m only in a few exceptional cases.

The alternative yielding principle, employing TH steel sets with sliding joints, is a method which has been customary in mining for decades. Haulage roads with diameters of 6–8 m and with high covers of 1,000 m or more have been headed in this way since the 1930s. Larger diameters have been rare.

In the case of the north drive in the Sedrun section, it quickly became clear that the great cover of 800 m in the Tavetsch intermediate massif north coupled with the large necessary excavation diameter of maximum 13 m would be the decisive factors in the development of a suitable innovative construction concept. Extensive





Credit: H. Ehrbar

▶ **Figs 8a and b** *The basic principle of excavation support for the squeezing rock of the Tavetsch intermediate massif north*

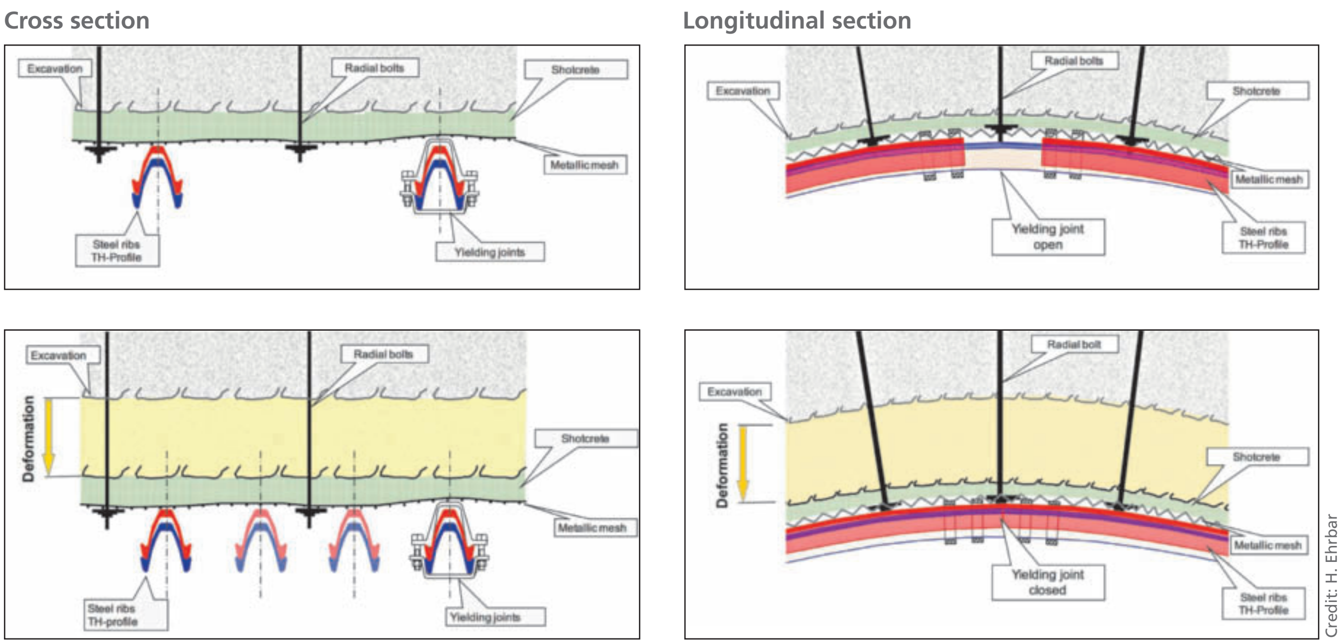
Forward exploration	Core boreholes, length: 31–196 m
Section	Round
Excavation cross section	81–134 m²
Additional excavation for deformation	Up to 75 cm radial deformation
Excavation support	Systematic radial anchoring: self-drilling grouting anchors; length: 8–12 m; ultimate breaking load: 320 kN; 96–288 m/rock bolts per metre of tunnel
Face stabilisation	Self-drilling grouting anchors; length: 12–18 m; ultimate breaking load: 320 kN; installed every approx. 6 m, up to 210 m/rock bolts per metre of tunnel; face sealed using shotcrete after each round of advance
Roof support	Self-drilling rock bolts as spikes; length: 6 m; ultimate breaking load: 320 kN; up to 100 m/rock bolts per metre of tunnel
Maximum support resistance	Theoretically approx. 2.0 MPa (without shotcrete), effectively approx. 1.0 MPa
Shotcrete	35–50 cm in the rear area, applied after dissipation of deformation

▶ **Table 1** *The tunnelling concept for the Tavetsch intermediate massif north [10]*

tunnel-structural analysis investigations and the interchange of experience with the Italian projects (Prof. Pietro Lunardi) and the German coal-mining industry made it possible to develop a highly promising solution. This was based in principle on the use of tried-and-proven technology, and on the following elementary assumptions:

1. Selection of a circular excavation profile, which would be best capable of absorbing the assumed hydrostatic pressures;
2. Selection of an appropriate degree of a larger excavation profile, in order to permit controlled deformations and thus a reduction of the necessary support resistance to a technically and economically justifiable level;
3. Only full-face tunnelling; it was possible here to benefit from the full-face excavation method, combined with face support by means of long rock bolts, which was, at precisely this time, being developed to perfection in Italy [6];
4. Clear assignment of the use of support materials with high deformation capacities (steel support and rock bolts) in the "deformation phase" and with low deformation capacities (shotcrete, concrete) in the "resistance phase"; TH steel set support combined with long rock bolts, known from mining, were used as the yielding support elements.

▶ **Fig. 8** shows the support concept selected for the Tavetsch intermediate massif north. It includes systematic extra excavation of up to 0.7 m for absorption of convergence, overlapping anchor bolting of the face, sealing of the excavation face using shotcrete and mesh, the TH steel sets and radial anchoring. Two complete rings of TH steel sets of eight individual segments in each case were fitted into each other in order to assure the high convergence necessary.



▶ **Figs 9a–d** *The rock support concept for the Tavetsch intermediate massif north: system as installed (top); condition after deformation (bottom)*

The most heavy-duty support type used for the project was the installation of a maximum of three TH steel set rings per metre of tunnel, extra excavation of 0.7 m and a total length of the radial anchor bolts of approximately 288 m per metre of tunnel [8]. It was not possible to conclusively determine during the project-planning phase the effective load-bearing performance of the radial anchor bolts under the large rock displacements forecast.

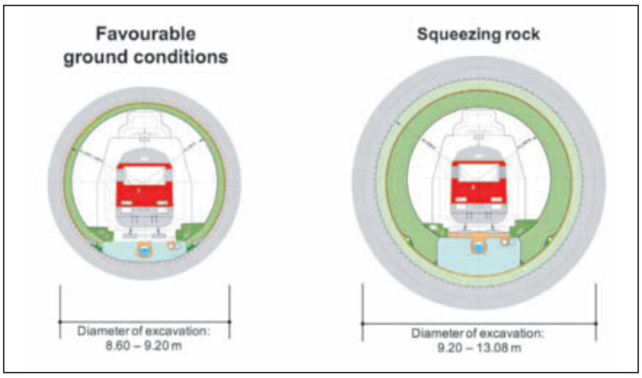
After the occurrence of the convergence corresponding to the extra excavation, the segments of the inner and outer steel set arches made contact with each other, thus increasing resistance. A 0.6 m thick shotcrete lining was planned at a distance of approximately 75 m behind the face, in order to preclude further convergence [8].

The maximum thickness of the inner lining to be installed after a long time delay was 1.2 m in the project phase. It was nowhere necessary to actually use this during construction, however, and the maximum thickness installed was instead 90 cm.

3 PREPARATION FOR CONSTRUCTION – OPERATIONAL CONSTRUCTION FEASIBILITY

Not only the rock-structural analyses, but also the demonstration of operational construction feasibility, were a central requirement during the early project phases. A comparison of the cross section for the heaviest type of support and that in stable rock clearly showed the unusual problems which would have to be overcome with respect to operational construction feasibility; that is, the accomplishment of an excavation section of 13 m in diameter in extremely squeezing rock.

A theoretically safe excavation support concept which could not, however, have been implemented at any economically rational level of input would not have been a usable solution. As the following plausibility observation shows, this risk of a safe but ultimately non-cost-effective excavation concept was not a mere spectre: the project engineer's comprehensive analyses of cycle times led to the conclusion that only just half of the rate of advance of 1.8 m/WD (WD = working day) [1] assumed in earlier studies would, at an average rate of tunnelling of approximately 1 m/WD, be possible. A further reduction in average rate of advance to 0.8 m/WD would have cost around a year of additional construction time. Similar, and even lower, rates of advance were also documented for various tunnel structures in squeezing rock [3]. The assurance of an efficient "industrialised" steel set arch support system was therefore another basic requirement for demonstration of the technical feasibility of the solution selected.



Credit: ATG

Fig. 10 *Comparison of standard cross sections for structurally favourable and squeezing rock*

The tunnel-engineering aspects could also be correspondingly analysed in more depth, thanks to the broadly founded expert knowledge available within the Construction Technology Working Group. A modular system with a minimum spacing interval of 33 cm was ultimately defined for the excavation support for mastery of the highly variable geological conditions (see **Figs 9a–d**).

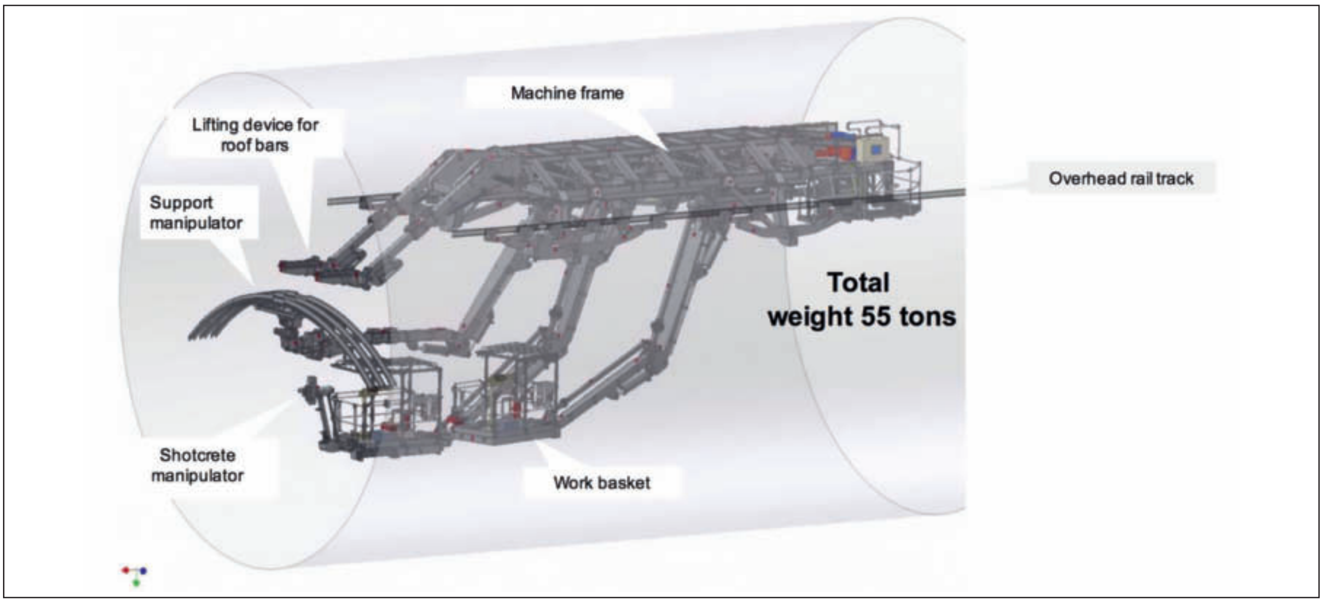


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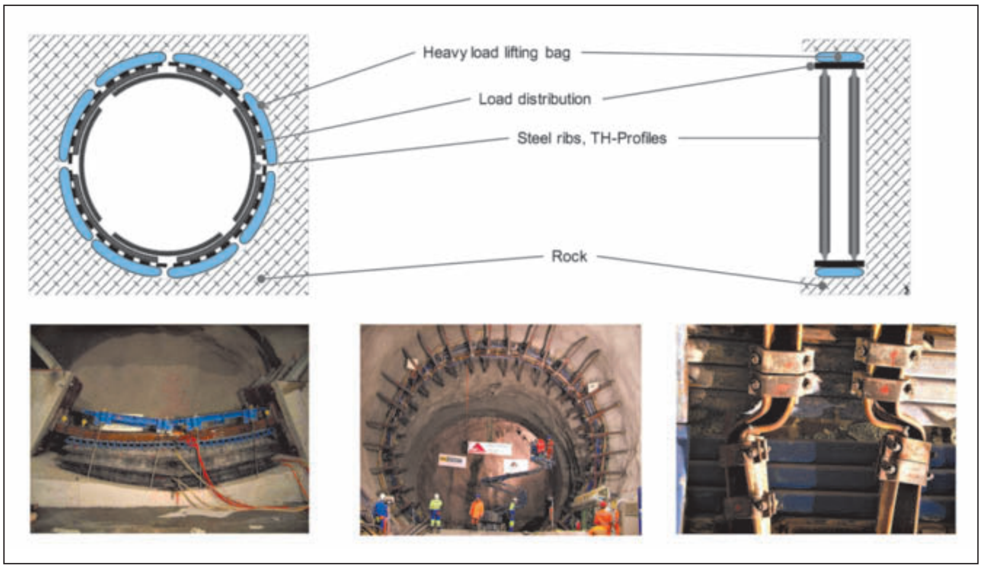
Fig. 11 *Rock support installation equipment as per preliminary studies*



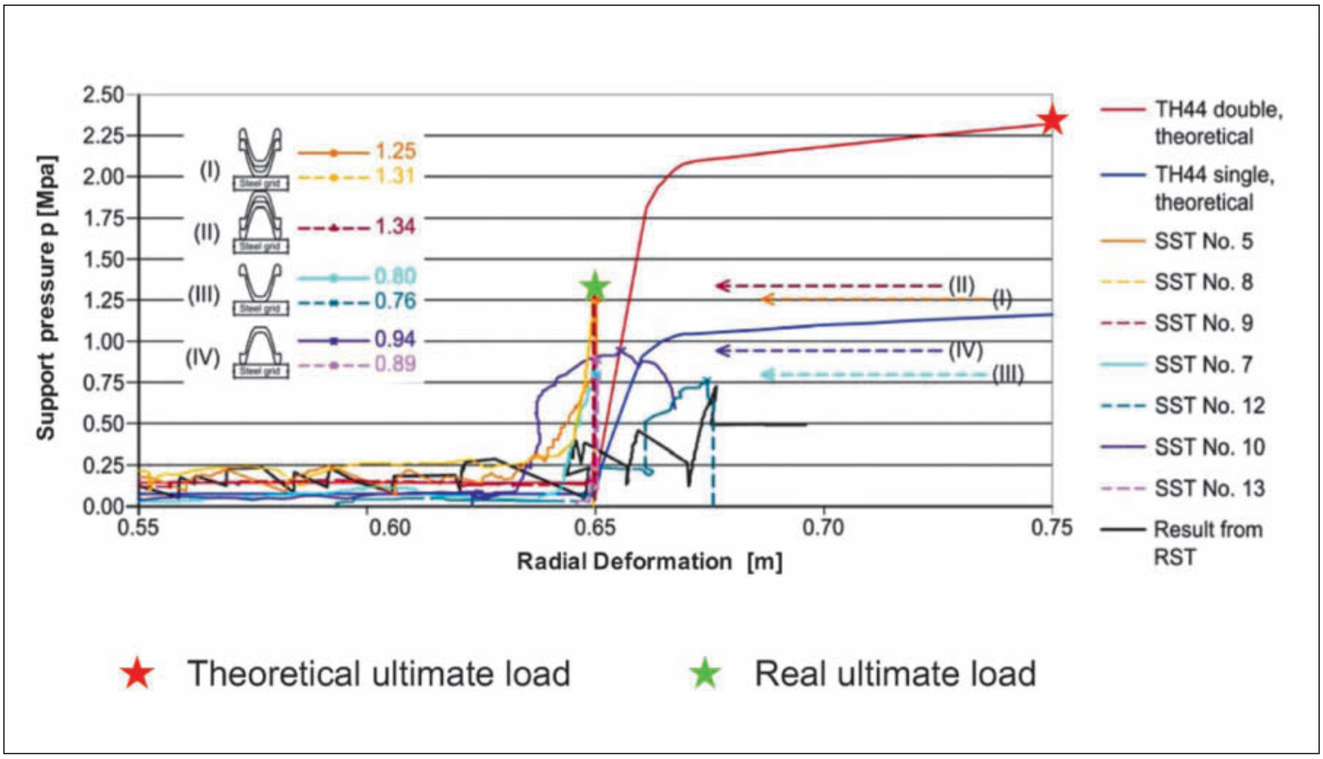
Credit: IG GBT Süd

▶ **Fig. 12** *In situ tests on steel support*

In addition to the interchange of knowledge with tunnel sites facing similar tasks in Italy and France, transfer of knowledge was also consciously maintained with, in particular, the German coal-mining industry. Visits to various coal mines demonstrated the importance of the interaction of the excavation support and the machinery and equipment used. The range of special machines employed in mining was most certainly also suitable for tunnelling applications. The mining machinery was used for significantly smaller excavation cross sections, however. To exclude any risks, the project engineer commissioned a feasibility study for a "rock support machine" from an equipment supplier specialising in the development and design of mining machinery prior to the drafting of the tender documents.

Feasibility was demonstrated with adherence to the high safety standards customary in the German mining industry, although the machine's dimensions exceeded anything previously known in mining. The rock support machine planned made it possible to suspend a major portion of the equipment on overhead monorails, in order to conserve and optimally use the constricted space available on the floor. This created significantly improved transport surfaces which, on the one hand, had a beneficial effect on rates of advance and, on the other hand, also achieved conditions more favourable to work safety, in particular.

Those project leaders were aware that the selected solution entered unexplored territory, since it was not possible to make use of any direct parallels in international tunnelling. The need for an extremely cautious procedure was therefore indicated. It was thus resolved, inter alia, to verify the performance of the steel sets for this unusually large section and under these great loads by means of tests. A decision was taken to perform these



Credit: IG GBT Süd

▶ **Fig. 13** *Results of in situ tests on steel support [8, 11]* load tests in situ in a niche in the rock, rather than in a test hall. Water-filled heavy-duty pressure cushions served to induce the deformations and simulate the rock pressure. The test apparatus, with a radial loading of up to 3 MPa and a maximum convergence of 0.7 m, is shown as used in schematic form in ▸ **Fig. 12** [9, 11].

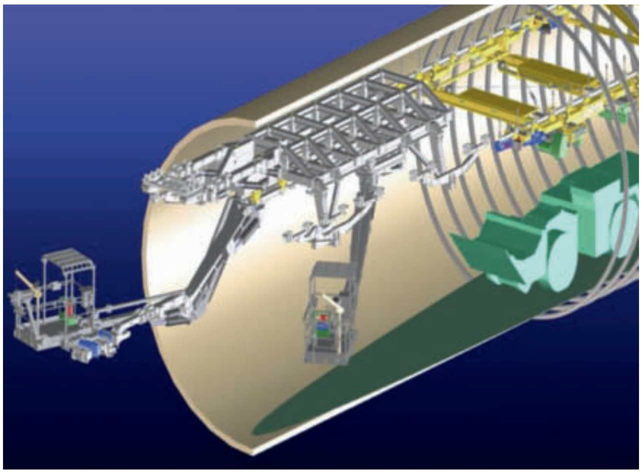
▸ **Fig. 12** shows a view into the 13 m diameter test niche. The surrounding rock was required to absorb the reaction from the application of load. The failure mechanism of the excavation support was characterised by buckling of the steel set arches.

The most important finding of these tests was the fact that the steel set arches were already fully buckled before completion of their total sliding path; they had not reached their full theoretical load-bearing capacity (see ▸ **Fig. 13**). The installation of additional steel set arches or a load-bearing shotcrete lining (after completion of deformations) was considered as a means of balancing out this deficiency if necessary.

The solution defined for tunnelling through the Tavetsch intermediate massif north had a scientific basis, with the tunnel structural-analysis discoveries providing substantiation of technical feasibility. It was not possible, by contrast, to demonstrate the feasibility of a tunnelling concept based on the top heading, since it was not possible to determine an equilibrium for the bottom-heading zones using the assumed boundary conditions. The client nonetheless decided also to permit the drive of top heading during the bidding phase. Bidders offering such an alternative would have had to supply the corresponding rock-structural analyses. No complete contractor variants were submitted during the bidding phase, despite the efforts of individual bidders to find an alternative.

4 CONSTRUCTION PHASE

The severely squeezing rocks of the Tavetsch intermediate massif north were reached in early 2004. The tunnelling system was then quickly optimised in such a way that it was possible



Credit: ROWA

▸ **Fig. 14** *The contractor's installation concept*

to advance a constant cycle of 1.34 m, rather than 1.00 m, per day. This rate of advance was also maintained in the zones of extremely severe deformations. The contractor used the following equipment in each tunnel to achieve these rates:

» A hydraulic excavator with a special boom, rotating head and pneumatic jackhammer for detachment and excavation of the rock;
» A Tamrock Axera T12 tunnelling jumbo, with four arms, for drilling and setting of the face and radial anchors;
» An articulated mobile loader with a 21 t all-up weight for outward haulage of the excavated material;
» A GTA rock support machine with a 56 t total weight, suspended on overhead monorails, for installation of the steel set arches and cutting of the face bolts to length;
» A shotcreting unit for application of the shotcrete support;
» A suspended mobile crane with a load-bearing capacity of 20 t for the handling of materials.

The overhead crane and the rock support machine for installation of steel support were key elements in the tunnelling cycle and ultimately permitted industrialised production of the excavation support system.

Credit: ATG

▸ **Figs 15a and b** *Excavation and immediate sealing of the face (left); installation of the face bolts using the Jumbo Axera T12 (right)*

Credit: ATG

▶ **Figs 16a and b** *The rock support installation equipment in use (left); cutting of the face bolts to length (right)*

The following experience was gained during tunnelling: mechanised excavation of the severely fault-gouged material generally caused no problems, although excavation was, at some points, of extremely small elements (see ▶ **Fig. 15**, left).

Installation of the steel support segments and the self-drilling rock bolts was also accomplished on schedule. The contractor had decided in favour of the use of steel rock bolts for face support (GRP rock bolts would also have been permitted), and these therefore had to be cut to length for every excavation round.

▶ **Fig. 16**, left image, shows the use of the suspended GTA rock support machine for the steel support installation.

The breakthroughs to the adjoining Amsteg section on 17 October 2007 in the west tunnel and 29 November 2007 in the east tunnel marked the conclusion of the excavation work.

The average daily advance rates per month achieved in the east and west tunnels are shown in ▶ **Fig. 18**. After an initial learning phase in early 2005, continuous working cycles were soon achieved and resulted in almost constant rates of tunnelling advance. Average advance rates agreed in the works contract was 1.1 m/WD. At an average of 1.04 m/WD, this was, in fact, almost achieved.

Credit: ATG

▶ **Fig. 17** *17 October 2007: breakthrough in the west tunnel*

Thanks to the construction method selected and to the better structural behaviour of the Clavaniev zone, it nonetheless proved possible to complete the excavation work considerably sooner than specified in the contractually agreed construction schedule, and at lower cost than originally envisaged. Despite convergences of up to 80 cm, it was also possible to avoid costly re-profiling as provided for in the terms of the contract.

5 GROUND BEHAVIOUR AS OBSERVED

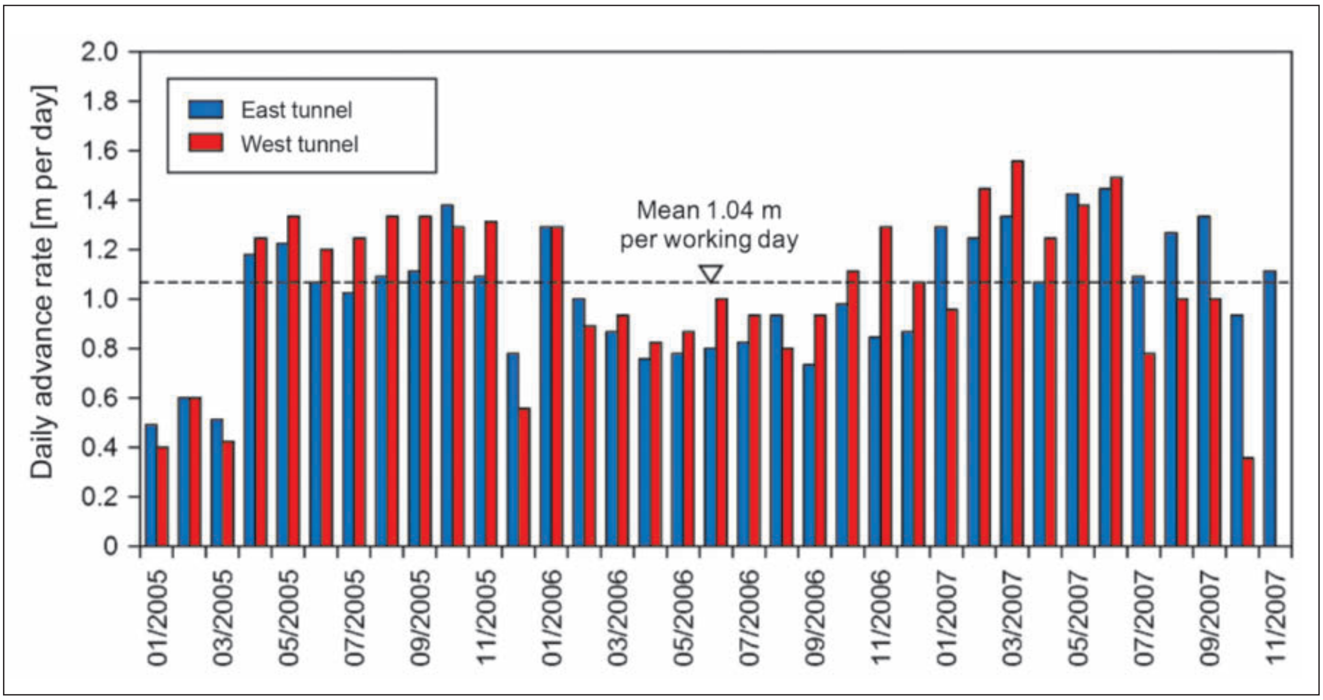
Average radial deformations in the squeezing zones were approximately 30–40 cm, and locally up to 80 cm. As expected, the deformations occurred not symmetrically, but instead highly asymmetrically. They were stopped within the permissible limits in all cases, however.

The successful control of tunnelling had also the advantageous effect that the re-profiling provided for in the contract was never needed. This fact attests to the excellence of the responsible geologists and site managers, who learned to "handle" the rock encountered within an extremely short time and deployed the corresponding provisions for excavation diameter and excavation support.

Tunnel metre 1,174 in the west tunnel is a good example to illustrate the typical structure behaviour as observed.

Oversize excavation at this point was 0.7 m. The face bolts had a length of 12 m, with an overlap of 6 m. Total length of the radial anchor bolts installed was 120 m rock bolts per tunnel metre, and one TH steel set arch was installed per tunnel metre. This was thus a relatively modest support resistance, but a large oversize excavation to accommodate rock deformations.

Deformations of up to 0.75 m were measured for the approximately uniform radial convergences which occurred. The

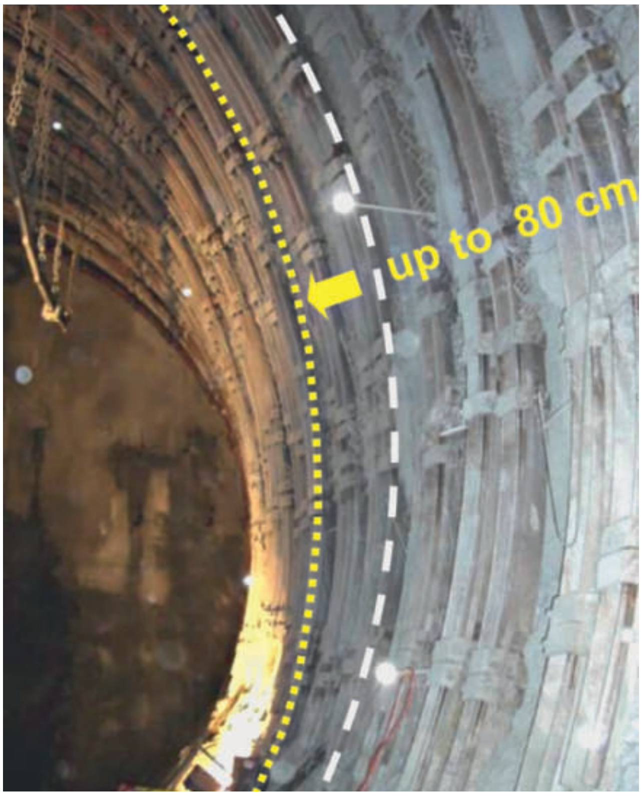


Credit: [9]

▶ **Fig. 18** *Average daily advance rates per month in the east and west tunnels [9]*

course of convergence, as a function of distance from the face, which was typical for the entire Tavetsch intermediate massif north, is worthy of note. It can be seen that more than 80 % of convergence occurred at a distance of one tunnel diameter, and that the major long-term deformations feared in this material did not take place.

The connections of the TH set sets also exhibited more or less regular closure. Unsurprisingly, given such deformations, the shotcrete sealing layer failed locally at some points, depending on stratification and foliation. Special high-strength steel mesh (so-called composite mesh) was indispensable as overhead protection for this reason (see ▶ **Fig. 21**).



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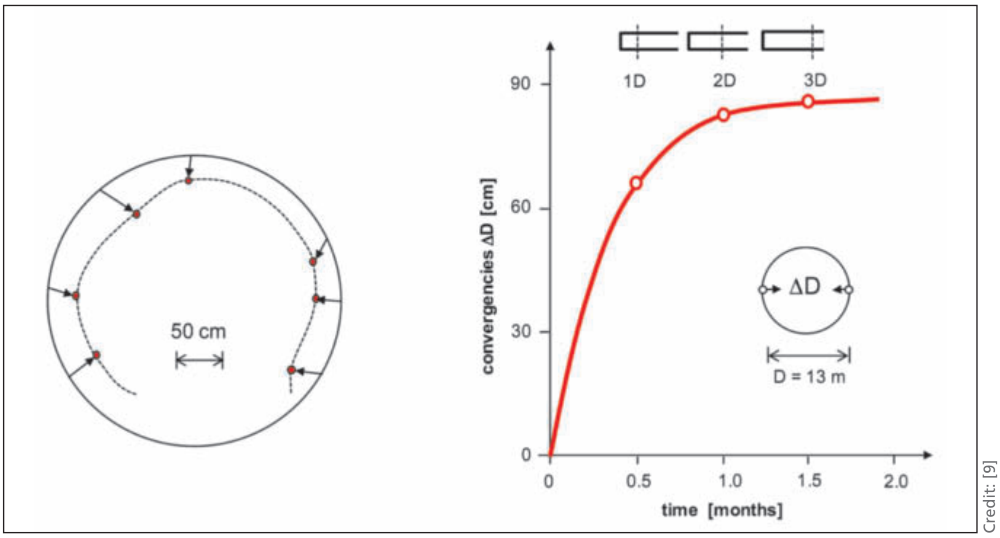
▶ **Figs 19a and b** *Deformations in the steel support*



▶ **Fig. 20** *Typical deformation process at tunnel metre 1,174*

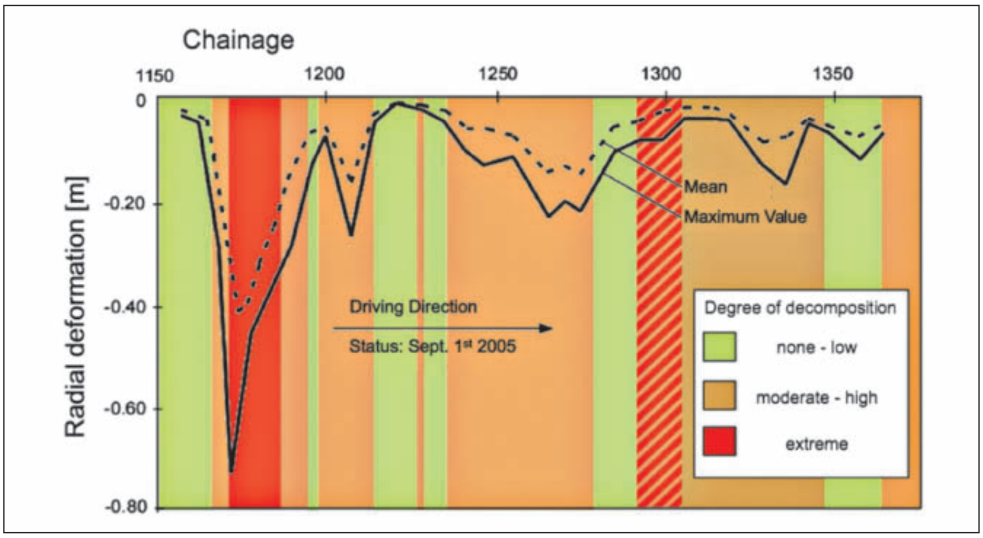
The highly alternating fault-gouged content of the rock and the presence of solid rock zones had very great effects on the overall convergence picture along the tunnel axis. ▶ **Fig. 22** shows a selected section of tunnel of approximately 200 m in length for which the correlation between the fault gouge content of the rock and measured convergence is shown. This is the plot of data averaged in the tunnel profile and that of the peak values. The mean figures for radial convergence vary between 10 cm and 40 cm, whereas local convergences of up to 75 cm occur in this section of tunnel.

6 CONCLUSIONS

Contrary to the fears of various experts, tunnelling in the Tavetsch intermediate massif north was ultimately conducted and completed successfully. Against what can the "stipulated" success be measured? The meeting of the quality, completion and cost targets is with certainty primarily definitive. Here, the following balance can be drawn:

» The completion target was met, and even bettered, with breakthrough occurring nine months earlier, thanks to better conditions encountered at the transition to the Aar massif.

▶ **Fig. 21** *Severely deformed lower side wall of tunnel, showing steel distortion mesh as overhead protection*



Credit: [9]

Fig. 22 *Rock load-bearing behaviour from tunnel metre 1,150 to tunnel metre 1,350 [9]*

» Only slight fluctuations in rates of advance occurred, despite greatly varying ground conditions.
» The cost objectives were met, and the client actually benefited from the contractually agreed shorter provision times for the installations for the tunnel heading in the Tavetsch intermediate massif north.
» Nowhere along the in total 2 km long tunnel-heading operations in squeezing rock was re-profiling necessary.
» Despite ultra-difficult ground conditions, there were no serious accidents in the Sedrun northern tunnelling operations.

The following factors made this success possible:

1. The integrally consistent tunnel structural concept may be mentioned as the primary success factor.
2. Of the greatest importance, however, is the fact that the contractor tackled the challenge of a totally new construction method. He assured continuous advances with the high degree of mechanisation used.
3. Thanks to systematic forward exploration and continuous observation and interpretation of the current ground behaviour, the project geologists at all times provided correct forecasts, appropriate to practice, of the upcoming ground behaviour.
4. The support of the engineer, the differentiated hazard scenarios defined by him, the balanced surveying system, the site management and expert support all, in their own way, contributed to success.

An essential element leading to success in the north tunnelling operations Sedrun is, however, probably also the fact that it proved possible, in a process lasting several years, to develop a construction method appropriate to local conditions which was consistently supported and optimally implemented by all those participating in the project (project engineers, geologists, local site management, crews, the contractor's workforce, the client and the experts). And, at no time did discussion of payments obstruct the taking of the necessary technical decisions.

The coping with the squeezing-rock section in the Tavetsch intermediate massif north may thus also be considered an example of partnership practised live on-site for the accomplishment of an exceptionally difficult task.

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